Long-Term Structural Health Monitoring of the San Ysidro Bridge (RoadLife)

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Bridge Research Center
New Mexico State University
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LONG-TERM STRUCTURAL HEALTH MONITORING OF THE SAN YSIDRO BRIDGE

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PREFACE

This paper discusses an initial live-load test on the San Ysidro Bridge as well as a subsequent load test on a full-scale single test bridge.

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ABSTRACT

As part of the Lifecycle Innovative Financing Evaluation (Road LIFE) initiative, the San Ysidro Bridge along US Route 550 will be monitored for ten years to determine changes in deflection, stiffness, and load-carrying capacity. This paper discusses a live load test on the San Ysidro Bridge as well as a subsequent load test on a full scale single lane test bridge. The two tests were used to determine the load rating for both shear and moment of the San Ysidro Bridge. The results of the load tests were compared with three finite-element modeling schemes. The modeling schemes consisted of frame, shell and solid elements to model the various bridge components. It was determined that a model using frame elements for the girders and shell elements for the bridge deck was much simpler to develop and produced nearly the same results as the more complicated models. This finite-element model was used to determine a load rating for the bridge. This load rating was then compared with the load rating using the distribution factors from the AASHTO Standard and LRFD Specifications. According to both AASHTO Specifications, the interior girder shear controlled the load rating of the San Ysidro Bridge. Using the finite-element modeling scheme of frame and shell elements the interior girder shear was found not to control, instead the interior girder moment controlled. This load rating will be used as a baseline for comparison with future load ratings throughout the warranty period.
ACKNOWLEDGEMENTS

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INTRODUCTION

Determining a reduction in a bridge’s load carrying capacity as it ages over time is a complex task. The moment and shear demand for a particular bridge girder depends not only on the magnitude and location of the applied load, but also on the condition and properties of the various bridge components. The AASHTO Manual for Condition Evaluation of Bridges (2000) provides guidelines on how to evaluate a bridge’s capacity through the use of inventory and operating ratings. Because of the complex nature of load distribution, engineers often times rely on codes (e.g. AASHTO LRFD Specification 1998 and AASHTO Standard Specification 1996) to provide simplified procedures for determining distribution factors to be used in determining the inventory and operating ratings. While these procedures have facilitated the capacity analysis, researchers (Barr et al. 2001; Chen and Aswad 1996; Burdette and Goodpasture 1988) have found the equations to calculate the distribution factors to be at times excessively conservative. In addition, incorporating any changes in the structural condition of the various bridge components into the load rating is challenging.

In December, 2001, the three-year reconstruction of US Route 550 in New Mexico was completed. This reconstruction included expanding the 118 mile stretch of highway from a narrow two-lane road into a four-lane highway within three years. A New Mexico State Highway and Transportation Department (NMSHTD, currently New Mexico Department of Transportation) initiative that included innovative highway financing, procurement, and warranty provisions were used to provide incentives to reduce the construction time (Albright et al. 2001). As part of this Road Lifecycle Innovative Financing Evaluation (Road LIFE) initiative, the pavement and bridge
performance will be maintained under warranty by the private sector consortium that
designed and built the roadway improvement.

Throughout the warranty of US Route 550, researchers from New Mexico State
University (NMSU) have been contracted to monitor the bridges for deterioration and
reductions in load carrying capacity. The researchers will monitor global bridge
parameters such as deflection, stiffness, and load-carrying capacity. Nondestructive
technologies such as diagnostic load testing, close range photogrammetry and Quick
Time Virtual Reality are being used to measure and document the global bridge
parameters. This paper focuses on the determination of the moment and shear load
carrying capacity for the San Ysidro Bridge. This bridge is the first in a series of bridges
to be tested along US Route 550 in order to determine baseline data that will
subsequently be used as a comparison throughout the warranty period.
SAN YSIDRO BRIDGE

The San Ysidro bridge was designed in the late 1970s by the New Mexico State Highway and Transportation Department (NMSTHD). The bridge was designed as simply supported for girder and deck dead load and three-span continuous for live load and superimposed dead load. The first and third spans are 13.7 m (45 feet) long, while the center span has a length of 13.4 m (44 feet). The original bridge was 15.5 m (51 feet) wide and was designed to carry two lanes of traffic with a 15° skew. Six AASTHO type II girders were used to support the original deck. Figure 1 shows an outline of the original cross sectional view of the San Ysidro Bridge.

![Diagram of San Ysidro Bridge Cross Section]

FIGURE 1 Cross section of the San Ysidro Bridge.

As part of the US Route 550 reconstruction project, the bridge was redesigned to carry four 3.7-m (12-foot) wide traffic lanes with two 2.4-m (8-foot) wide shoulders in each direction. A 1.8-m (6-foot) wide median was used in the center of the bridge to separate the oncoming traffic. An 810-mm (32-inch) high jersey barrier was placed on each side of the bridge. As a result, the overall width of the redesigned bridge was
22.3-m (73-feet) wide. Figures 1 and 2 show a cross-sectional and elevation view of the widened San Ysidro Bridge respectively.

**FIGURE 2  Elevation view of the San Ysidro Bridge.**

Eight total (two new), AASHTO Type II precast, prestressed AASHTO I-Beams are used to support the bridge deck. The 28-day design compressive strength of these girders was 37.9 MN/m² (5.5 ksi) with a release strength of 32.7 MN/m² (4750 psi). Twelve 12.7-mm (½-inch) diameter low-relaxation prestressing strands are used to help support the loads. Ten straight strands were placed at the bottom of the girder in two rows and two straight strands were placed in the web. The overhang distance from the edge of the bridge deck to the centerline of the exterior girders web is 1.30 m (4.25 feet). The interior girder spacing varied for the new and old portions of the bridge. The spacing from the exterior girder to the first interior girder was 2.97 m (9.75 feet). The spacing of the remaining interior girders was designed to be 2.74 m (9.0 feet) (Figure 1).
The reinforced concrete bridge deck had a 28-day design compressive strength of 20.7 MN/m$^2$ (3.0 ksi). Because the interior girder spacing for the San Ysidro Bridge varied, the deck had two separate thicknesses. For the original portion of the bridge, the thickness of the deck was 203 mm (8 inches). The deck thickness was increased to 241 mm (9.5 inches) over the exterior girders and the first interior girders for the new portion of the bridge.
LOAD TEST

In order to determine the in-situ load carrying capacity of the San Ysidro Bridge, a diagnostic load test was performed. Externally mounted, Bridge Diagnostic Strain Gages were used to monitor changes in strain that the girders experienced as a load truck was driven across the length of the bridge. Strain gages were attached at the midspan of Span 3 girders. Instrumenting the Span 3 girders was chosen since it produced the critical bridge moment as well as it was the easiest to instrument. At midspan, three strain gages were placed at different elevations along the height of each instrumented girder. One gage was attached to the bottom flange of the girder to record the largest expected changes in strain. The second and third gages were placed at the bottom and top of the web respectively.

A three axle, 316 kN (70.9 kips) water truck was used to apply loads to the bridge. This truck was supplied by NMSTHD and was the heaviest truck that was available at the time for the load test. The axles of the truck were weighed prior to the load test. The front, 2nd and 3rd axles of the truck were tandem and weighed 56.6 kN, 123 kN, and 136 kN (12.7, 27.7, and 30.6 kips respectively). The distance from the front axle of the truck to the second axle was measured to be 5.3 m (17.5 feet). The distance from the second axle to the third axle was 9.9 m (32.5 feet). The width between the left and right tires was 2.0 m (6.67 feet).

Three load paths were chosen to apply the truck load to the bridge. The 1st load path \((Y_1)\) was located at a distance of 0.3 m (13 inches) from the interior face of the jersey barrier to the center of the right front tire of the truck. This load case was intended to apply the majority of the truck load to the exterior girder. For the 2nd load path \((Y_2)\),
the right front tire of the loading truck was placed at a distance of 3.2 m (10.5 feet) from the interior face of the Jersey barrier. This distance was intended to apply the load to the first interior girder. For the last load path (Y₃), the right front tire was placed at a distance of 5.6 m (18.5 feet) from the interior face of the Jersey barrier. Furthermore, this load path placed the load over the third girder. Figure 1 shows the lateral distances of the three load paths on a cross sectional view of the bridge in relation to the six instrumented girders.

Changes in strain were recorded for each instrumented girder as the truck was driven along the three load paths. The truck load was applied to the bridge by driving the truck along a given path at a rate of 5 to 10 miles per hour. The slow traveling speed was necessary in order to reduce any dynamic effects of the live load which may be recorded by the strain gages. The truck was driven along each load path three times in order to verify consistency. Once the strain data was collected for each of the load paths, girder moments were calculated based on mechanics and design material properties.
FIGURE 3 Moment influence line for girder 1 and load path Y₁.

Figure 3 shows the moment influence line in relation to the truck front axle for Girder 1 (exterior girder) when the truck is driven along load path Y₁.

Figure 3 shows that the moment influence line for Girder 1 vary from positive to negative moment depending on the truck location. The first portion of the graph is approximately zero because the truck is moving along the first span and no significant strain values are recorded on the third span where the strain gages were placed. As the truck entered Span 2, the moment in Span 3 is negative as expected. Figure 3 also shows the moment values raising, decreasing, and then raising again as the truck traveled longitudinally along Span 3. This is due to the fact of where the axles are in reference to the strain gages. For example, the first maximum in the figure is where the truck is positioned so that the strain gages on the third span are in between the front and center
axles of the truck. The measured moment then decreases as the front and center axle move off of the bridge. The last maximum is where the back axle of the truck is approaching the strain gages location on the third span. The measured moments then decrease to zero as the water truck exits the bridge. Although the magnitude of load for Path 1 produced the largest moment in the exterior girder, similar trends were recorded with the other instrumented girders and for each of the load paths.

**Finite-Element Comparison**

After the load test, a finite-element model of the San Ysidro Bridge was created using a modeling scheme of shell and frame elements. This modeling scheme has been shown to be relatively simple and accurately model the behavior of slab-on-girder bridges (Barr et al. 2001; Mabsout et al. 1997). The concrete bridge deck was modeled using shell elements. The nodes for the shell elements were implemented at 0.15-m (0.5-foot) intervals along the width and 0.30 m (1-foot) intervals along the length of the bridge. This fine mesh was deemed necessary to accurately place the wheel loads on the bridge. Six degree-of-freedom space frame elements were used to model the precast, prestressed AASHTO I-Beams. The frame nodes were placed at 0.30-m (1-foot) intervals longitudinally along the bridge and laterally at the respective girder location. Vertically, the shell and frame elements were placed at the centroids of their respective members and rigid links were used to connect the two elements. These rigid links ensured member compatibility and that plane sections remained plane.

In the finite-element model of the San Ysidro Bridge, a skew angle of 15 degrees was used to simulate the actual bridge. Furthermore, in order to model differences in deck thickness between the original bridge and the widened bridge, the bridge deck was
broken down into nine different sections (three per span). This was done in order to accurately account for the different modulii of elasticity for each section as well as the different thicknesses of the bridge deck.

Frame elements were also used in order to model the 4.9 m (16 foot) columns that were used at the interior supports of the bridge (Figure 2). Rigid links were then used to connect the columns to the girders and the girders to the concrete deck. Figure 4 shows a superstructure cross section of the finite-element modeling scheme that was created to simulate the San Ysidro Bridge.

![Finite-element model of the San Ysidro bridge.](image)

The wheel loads of the water truck that was used during the load test on the San Ysidro Bridge was applied as individual concentrated loads on the finite-element model. The simulated truck load that was placed on the finite-element model was positioned so that the right front tire was along the respective load path (Y₁, Y₂, or Y₃) that was being investigated. Where the point load did not coincide with a nodal location, shape functions were used to distribute the concentrated load to the four shell nodes. The simulated truck load was positioned at different longitudinal locations along the
spans with an incremental spacing that varied between 1.5 and 3.0 m (5 and 10 feet). The reason for the smaller distance of 1.5 m (5 feet) was to obtain a finer mesh at locations of critical moments. The incremental spacing for the longitudinal truck loading was similar for each of the load paths $Y_1$, $Y_2$, and $Y_3$.

The material properties of the finite element model were obtained based on an optimization approach. The original portion of the bridge had been in service for many years and it was believed that over-weight truck loads caused the pier cracking which would reduce the rotational stiffness at the pier. This reduction in rotational stiffness was taken into account by reducing the modulus of elasticity of the original portion deck and girder. After a parametric study, the original portion of the bridge deck was given an elastic modulus value of 16,200 Mpa (2350 ksi). The modulus of elasticity that was obtained for the girders which support the original portion of the bridge deck was 22,900 Mpa (3320 ksi). These stiffness values were used for all three load paths that were used to apply the simulated truck load. The new portions of the bridge were assumed to have the material properties based on the design values. Figure 3 shows a comparison of the Girder 1 midspan moment calculated using the finite-element model with the truck along Path 1. The finite-element moment has the same positive and negative moment trends as the load test moment. Overall, the finite-element moment was within 8% of the measured moment when the truck was in the third span of the bridge.

The measured and finite-element longitudinal moment values for all three load cases were also plotted against each other in order to show the comparison between the finite-element model and the load tests. A linear best fit line was created in order to
determine how closely the finite-element model compares to the three load tests (Y₁, Y₂, and Y₃).

**FIGURE 5 Comparison of measured and finite-element moment.**

Figure 5 shows all of the midspan moments obtained from the finite-element model for the three load tests plotted against the midspan moments obtained from the actual load test. A linear best fit line was calculated and is shown on the figure. The best fit line had a slope of 1.043 and a coefficient of correlation of 0.97 which indicated an overall good comparison between the measured and finite-element moments.

The finite-element moment was also compared to the load-test moment transversely over the width of the San Ysidro Bridge. In other words, when the truck was in the longitudinal location that produced the maximum moment (near the midspan of
Span 3), the measured and calculated midspan moments for each of the instrumented girders were compared for each load case. Figure 6 shows the comparison between the finite-element model and the load test transversely over the width of the San Ysidro Bridge for Path 2. The numbers 1 through 6 on this figure correspond to the six instrumented girders at the critical moment.

**FIGURE 6** Comparison of midspan moments for path 2.

Figure 6 shows that when the truck was at the critical moment, Girder 2 carried a majority of the truck load. Girders 1 and 3 carried a little less than half the magnitude of Girder 2. The remaining instrumented girders (4 through 6) carried almost none of the load. In comparing the three load cases, the maximum measured girder moments were on average within 5% of the finite-element moment.
SINGLE-LANE TEST BRIDGE

Because of the heavy traffic that this stretch of highway had experienced and because of the cracking at the supports, the NMSHTD was also interested in obtaining the load rating for the San Ysidro Bridge for shear. Less research has been done in the area of shear distribution (Ebeido and Kennedy 1994; Zokaie et al. 1991), but some agencies have found that shear has controlled the load rating of their bridges (Al-Mahaidi and Taplin 2000). Shear could not be directly measured on the San Ysidro Bridge during the live-load test, so a full-scale single lane, slab-on-girder test bridge located in the New Mexico State University Structural Research Lab was used to evaluate the effect of shear loads on I-girder bridges. The test bridge had a span length of 12.2 m (40.0 foot) long and was simply supported on jersey barriers. Three W530x92 (W21x62) steel girders that were salvaged from another bridge were used to support a 0.15-m (6-inch) thick reinforced concrete bridge deck. The width of the bridge deck was 3.4-m (11-ft). The overhang distance from the edge of the bridge deck to the centerline of the exterior girder was 0.3 m (1 ft). The interior girder spacing was 1.4 m (4.5 ft). Figure 7 shows a cross sectional view of the test bridge.

![Figure 7: Cross section of test bridge.](image-url)
In order to measure the girder reactions from the load test, three load cells were then placed underneath each of the W530x92 (W21X62) girders at one end of the test bridge. Once the bridge was lowered onto the load cells, initial readings of load were recorded. These self weight loads were subsequently subtracted from any changes in reaction due to the externally applied load.

A reaction frame was used to apply the concentrated deck loads during the test. This reaction frame could be positioned longitudinally along the first 4.9 m (16 feet) of bridge and anywhere along the width of the bridge. A 220 kN (50 kip) capacity ram, which was attached to the reaction frame, was positioned at various longitudinal and lateral locations along the deck.

During the test, a load cell was placed between the ram and the bridge deck to measure the applied force. The load was then applied by means of a manual jack. The concentrated load at each load location was sustained for approximately one minute. During this time readings were taken at 60 readings per minute. The measured reactions were nearly constant for all load locations and varied by only 1 % during the one minute. Figure 8 shows a picture of the reaction frame and the procedure of loading the bridge deck.

![Figure 8](image)

**FIGURE 8 Load test of single lane test bridge.**
After the load test was performed, three finite-element modeling schemes were used to model the test bridge in the structural research lab. All three of these finite-element models were created using the computer program SAP2000 Nonlinear (SAP 2000). The first model used shell elements to model the bridge deck and frame elements to model the girders. The second model used shell elements to model both the girder and the bridge deck. The last modeling scheme used solid elements to model the deck and frame elements to model the girder. The elements were placed vertically at the centroid of their respective members and connected through rigid body constraints. For all three finite-element models the deck was modeled with nodes at 152 mm (6 inch) spacing laterally and 305 mm (12 inches) longitudinally. This kept the aspect ratio constant at 2 to 1. The nodal spacing for the girders was positioned at 305 mm (12 inches) longitudinally to coincide with the deck elements. At the time of testing, no material samples of the bridge deck were available to accurately establish the modulus of elasticity. As a result, the compressive strength ($f_c$) was assumed to be equal to the design value of 27.5MN/m² (4000 psi) and the modulus of elasticity was calculated according to the equation listed in the ACI code (2002).

The measured loads that were placed on the test bridge were then applied to the three finite-element bridge models. The reactions for each of the girders due to the applied load was calculated. Figure 9 shows a comparison of the measured test girder shear forces for the three girders as well as the calculated shear forces from the three finite-element models.
FIGURE 9 Comparison of measured and calculated shear forces.

For Figure 9, the reactions from the bridge test are plotted on the x-axis and the finite-element model reactions are plotted on the y-axis. A linear best fit line and coefficient of correlation were also calculated and the results are shown for each of the three finite-element models. Overall, each of the finite-element models closely reproduced the measured shear values. All three finite-element models were within 4% of a perfect correlation (i.e. slope = 1.0) of measured and test data. The frame and shell model was somewhat stiffer than the other two models and on average produced slightly conservative results. Because this model was easier to implement and produced similar results to the more complicated models, the frame and shell modeling scheme was chosen to evaluate the load rating of the San Ysidro Bridge for both moment and shear.
LOAD RATING

Once the finite-element model for the San Ysidro Bridge was validated for both shear and moment, the model was used to establish a baseline load rating for the bridge. According to both AASHTO Specifications, an AASHTO HS-20 truck is used to apply the live load to a bridge. Prior to the AASHTO HS-20 truck being applied to the finite-element model, the longitudinal locations of maximum shear and moment for each span were determined. These locations were obtained by calculating shear and moment influence lines. At the critical longitudinal locations, the bridge was divided into as many 3.7 m (12-foot) wide lanes as possible (4) and the AASHTO HS-20 truck load was systematically moved within each lane.

The AASHTO HS-20 truck was initially positioned at 0.61 m (2 feet) from the edge of the curb at the critical longitudinal location and was moved at the 0.15 m (0.5-foot) increments along the width of the bridge. For the single truck loading case, the truck position varied from 0.61 m (2 feet) to 3.5 m (11.5 feet) from the edge of the curb. For the two simultaneous truck conditions, each truck was placed in their respective design lane and systematically moved at 0.15-m (0.5-foot) increments. In developing the various load cases, it was assumed that the truck could get no closer than 0.61 m (2 feet) from the edge of a design lane. Furthermore, one, two and three truck load combinations were considered with the finite-element model. The three truck load combination did not govern the design as the median prevented the third truck from being close enough to significantly influence the exterior or first interior girder. For the one and two truck load case, there was 20 independent truck locations considered for each longitudinal location.
The maximum shear and moment values were then obtained for the load cases. These maxima were then multiplied by their appropriate multilane reduction factors.

According to the AASHTO LRFD Specifications (1996), the maximum reaction values for the interior and the exterior girders must be considered independently. Once these maximum reactions are obtained for the interior and exterior girders considering all possible lateral load combinations, the distribution factors based on the finite-element model can then be calculated. Equation 1 shows how the moment distribution factors are calculated using the finite-element results.

\[
DF = \frac{M_{FE}}{M_{MAX}}
\]  

Where:  
\( M_{FE} \) = finite-element maximum girder moment for each span  
\( M_{MAX} \) = girder moment calculated using a single continuous beam

The procedure to calculate the shear distribution factor was the same as the moment, but maximum values of shear were used in Equation 1 instead of moment. Table 1 lists the calculated distribution factors for the interior and exterior girders based on the finite-element model of the San Ysidro Bridge. Also listed in Table 1 are the distribution factors calculated according to the AASHTO Standard Specification (1996) and the AASHTO LRFD Specification (1998).
TABLE 1  Live-load distribution factors for the San Ysidro bridge.

<table>
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<tr>
<th>Distribution Factor Location</th>
<th>Model</th>
<th>Standard</th>
<th>LRFD</th>
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<tbody>
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<td>Interior Girder Positive Moment (3rd span)</td>
<td>0.79</td>
<td>0.82</td>
<td>0.80</td>
</tr>
<tr>
<td>Exterior Girder Positive Moment (3rd span)</td>
<td>0.74</td>
<td>0.77</td>
<td>0.79</td>
</tr>
<tr>
<td>Interior Girder Negative Moment</td>
<td>0.72</td>
<td>0.82</td>
<td>0.80</td>
</tr>
<tr>
<td>Exterior Girder Negative Moment</td>
<td>0.75</td>
<td>0.77</td>
<td>0.79</td>
</tr>
<tr>
<td>Interior Girder Positive Moment (2nd span)</td>
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<td>0.82</td>
<td>0.80</td>
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<tr>
<td>Exterior Girder Positive Moment (2nd span)</td>
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<tr>
<td>Interior Girder Maximum Shear</td>
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<td>Exterior Girder Maximum Shear</td>
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After the finite-element distribution factors were calculated, a rating analysis was performed based on the Load Factor Design (LFD) Method specified in the AASHTO Manual for Condition Evaluation of Bridges (2000). A more detailed description of the rating analysis can be found in Najera (2003). Load ratings were calculated based on the distribution factors obtained at the critical locations on the bridge. At each location, a separate load rating was calculated for interior and exterior girders. Therefore, a total of eight different load ratings were calculated for the San Ysidro Bridge. The equation to calculate the load rating by this method is listed as Equation 2.

$$RF = \frac{\phi R_n - \gamma_D D}{\gamma_L L (1 + I)}$$  (2)

Where:
RF = bridge load rating factor (either operating or inventory)
\(\phi R_n\) = nominal member capacity
\(\gamma_D\) = dead load factor (1.3)
D = nominal dead load effect (including non-composite and composite dead load)
\(\gamma_L\) = live load factor

= 1.3 (for operating rating) and 2.17 (for inventory rating)
L = nominal live load effect (caused either by HS-20 truck or lane loading)

I = live load impact factor

The load ratings were performed using the distribution factors found from the finite-element model as well as those calculated using the LRFD and Standard Specifications.

**TABLE 2  Load rating values for the San Ysidro bridge.**

<table>
<thead>
<tr>
<th>Girder</th>
<th>Finite-Element Model</th>
<th>AASHTO Standard</th>
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<td>2.15</td>
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</tr>
<tr>
<td>Span 3</td>
<td><strong>2.09</strong></td>
<td><strong>1.25</strong></td>
<td>2.02</td>
</tr>
<tr>
<td>Exterior</td>
<td></td>
<td></td>
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<tr>
<td>Span 2</td>
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<td>2.80</td>
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<tr>
<td>Interior</td>
<td></td>
<td></td>
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</tr>
<tr>
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<td>1.85</td>
<td>2.63</td>
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<tr>
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<tr>
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<td>1.36</td>
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<tr>
<td>Exterior</td>
<td>2.23</td>
<td>1.33</td>
<td>2.17</td>
</tr>
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</table>

Table 2 shows the calculated truck load ratings for the shear on Span 3, maximum moment on Spans 2 and 3, and the negative moment at Pier 2 based on the AASHTO LRFD and Standard Specification and the finite-element model distribution factors respectively. The lane load was found to never control the load rating. For Table 2, the operating rating is a measure of the maximum permissible load that can be applied to the
structure, and the inventory rating is a measure of the maximum load level which can be safely applied to the bridge over the duration of the bridge’s life.

Table 2 shows that the load ratings are larger when using the finite-element distribution factor when compared to the AASHTO Standard or LRFD distribution factors. This is due to the fact that the distribution factors in AASHTO Standard and LRFD Specifications are inherently conservative and the finite-element model distribution factors are less so. According to Table 2, the maximum shear on the third span for the interior girder controlled the load rating for both the AASHTO Standard and LRFD Specifications. However, using the finite-element distribution factor for the interior girder shear raised the inventory rating by 13% and 23% for the AASHTO Standard and LRFD rating respectively. When the finite-element shear distribution factors were used in the load rating, the positive moment for the Span 3 interior girder controlled the load rating with an operating rating of 2.09 and an inventory rating of 1.25. This finite-element load rating will be used as a baseline for future comparisons.
CONCLUSIONS

As part of a study on the health monitoring of the bridges along US Route 550, the San Ysidro Bridge was instrumented and subjected to a live-load test. After the test, the truck loads were placed on a finite-element modeling scheme of frame and shell elements and the results were compared with the results of the live-load test. Because of the concerns due to pier cracking, this same modeling scheme was validated with the results of a shear-load test on a single lane test bridge at New Mexico State University. The finite-element model of the San Ysidro Bridge was then used to calculate the load rating for the bridge. The finite-element load ratings were compared with the load ratings calculated according to the AASHTO Standard and LRFD distribution factors. These load ratings will be used as a baseline rating throughout the warranty period.

The study lead to the following conclusions:

- Based on the load ratings using the AASHTO Standard and LRFD distribution factors, shear controlled the load rating of the San Ysidro Bridge with an operating rating of 1.2 and 1.11 respectively. However, when the finite-element distribution factors were used, the positive moment on the third span controlled the load rating with an increase in the inventory rating of 4% and 13% respectively.

- In general, the AASHTO Standard and LRFD distribution factors were close to the finite-element distribution factors. This closeness is especially encouraging since the bridge had been widened and consisted of variable girder spacing along with cracking at the piers. The distribution factors calculated based on the San Ysidro model were on average 8% smaller than the distribution factors
calculated according to the AASHTO Standard and LRFD Specification. Overall, the AASHTO Standard and LRFD distribution factors were closer to exterior finite-element distribution factors than the interior girder.

- On average, the midspan moments calculated using modeling scheme of frame and shell elements were within 5% of the experimental results based on strains recorded during a load test.

- Three modeling schemes using combinations of frame, shell and solid elements were compared with the results of a shear test on a full scale, single lane Test Bridge. A modeling scheme using frame elements for the girder and shell elements for the deck produced shear values that were on average 2.8% higher than the measured results. Using shell and solid elements to model the girder and deck respectively did not significantly improve the comparison between the measured and calculated results.
REFERENCES

American Concrete Institute (ACI). *Building Code Requirements for Structural Concrete (ACI 318-02)*, Farmington Hills, MI. 2002


